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Mr. Ralph E. Rodgers
Assistant General Counsel
Tennessee Valley Authority
400 West Summit Hill Drive
Knoxville, Tennessee 37902-1401

Re Root Cause Analysis
Failure of Ash Pond Containment Structure
TVA's Kingston Fossil Plant

Dear Mr. Rodgers:

This letter presents my conclusions concerning the root cause analysis of the reference failure. In accordance with my agreement with TVA dated January 29, 2009 the purpose of the agreement is to "provide advice and assistance in the nature of consultation and peer review in connection with the root cause analysis study of the failure of the ash pond containment structure at TVA's Kingston Fossil Plant, and other related matters".

Background

The Kingston fly ash and bottom ash disposal included dredge cells in which mostly fly ash was stored and placed by wet methods i.e. it was sluiced into place. The fly ash was contained by a series of dikes built by upstream construction methods. At the northern edge of the dredge cells there was a 200 ft wide setback between the initial perimeter dike C, and the subsequent series of dikes. The dikes were built of compacted fly ash, bottom ash and initially of natural silty clays. The total thickness of ash had reached about 85 ft at the time of the failure. The failure occurred in the early hours of December 22, 2008 releasing about 5.4 million cubic yards of ash that pushed one single-family home entirely off its foundation, and ultimately rendered three structures uninhabitable. It is estimated that as many as 42 residential properties may have been affected. The failing mass moved as far as 3,200 ft beyond the limits of the original ash pond and came to rest with a slope inclination of less than 0.5 degrees.

The root cause analysis (RCA) was performed by AECOM under the direction of Mr. William H. Walton. I worked closely with AECOM throughout all phases of the investigation. Thus, my input was considered and, if appropriate, incorporated into the various phases of AECOM's work, including field explorations, laboratory testing and analysis of the data.

The preparation of this letter is based on the information presented in a report prepared by AECOM, dated June 25, 2009 and entitled "Root Cause Analysis of TVA Kingston Dredge Pond Failure on December 22, 2008". In the preparation of this letter it was assumed that the reader is familiar with the AECOM report.

Scope of Work

I performed two site visits on January 20 and February 25, 2009 to inspect the failure debris and to observe the field exploration by AECOM that was in progress. The overall program and the methods that were being used in the field exploration were reviewed.

I also attended 5 meetings with AECOM and TVA personnel in the TVA offices in Chattanooga, TN, to review the progress of the RCA work and preliminary results. In addition I visited the AECOM offices in Vernon Hills, IL in three separate occasions to observe the soil samples being tested and review the results of laboratory testing, analysis and report preparation that were in progress. Undisturbed samples of the interface between the ash fill and the foundation soils were sent by AECOM for testing to the University of Massachusetts in Amherst, MA. Because of the importance of the interface of the ash and the natural soils, I visited the laboratory in Amherst twice to examine the samples after extrusion from the tubes and to observe the testing.

This letter addresses only the causes and mechanism of the December 2008 failure. The conclusions are based mainly on the findings and the results of analyses presented in the AECOM report. Previous engineering reports on the ash pond were reviewed for geotechnical data and for observations made prior to AECOM's work, but not for adequacy of previous designs, nor for conformance to standards of practice.

Discussion and Conclusions

The field explorations and laboratory testing performed by AECOM were extensive and achieved a comprehensive characterization of the subsurface conditions encountered after the failure, both in failed and unfailed areas of the dredge cells. A detailed survey of the failure debris found various objects with known pre failure locations and thus it was possible to estimate the direction of the movements of various elements of the dredge cells during the failure.

The foundation soils for the dredge cells consisted generally from the surface down of a) a thin (less than 6 in thick) layer of a laminated material consisting of thin (a few mm thick) laminas of what appears to be dark grey fine fly ash and reddish brown clayey silt, which is referred to in the AECOM report as slimes; b) soft to stiff clays; c) alluvial sand

and silts; and d) shale bedrock. The slimes and the clays were not found in some locations. From a geotechnical standpoint the weakest foundation material was the material referred to as slimes which would thus control stability for failures that may go through the foundation and thus it is discussed in more detail below.

The measured slimes properties are those of the combined laminas as testing of individual laminas would be difficult and for some properties not possible. From the gradation standpoint it is a silt, its particles are predominantly spherical indicating it is mostly ash, liquid limits range from 30 to 60 and plasticity indices range from 10 to 25. The slimes are unusual in that the natural water content is substantially higher than the liquid limit with liquidity indices ranging from 4.7 to 7, which means that if the slimes are remolded, they will be transformed into a liquid slurry. It develops the peak undrained strength at about 5% shear strain with subsequent decrease in resistance so that at 20% strain its resistance is about 75% of the peak. As noted above if fully remolded it becomes practically a fluid.

The sluiced ash is also primarily of silt size and its particles are mostly spherical. It has no plasticity. The in place condition of the ash is very loose with very low or weight of hammer penetration resistance in SPT testing. The friction measured in cone penetration testing is negligible. The median and average void ratios of the unfailed ash are 0.84 and 0.88 respectively, while the ash that failed and was subsequently redeposited is slightly denser.

Reconstituted samples were prepared at void ratios so that during shear strength testing the void ratios ranged between 0.75 and 0.94. When tested in drained triaxial tests for the range of voids measured in the field, the ash exhibits behavior typical of loose cohesionless soils with a friction angle of about 30 degrees and with a substantial decrease in volume during shear. In consolidated undrained triaxial tests it has a low strain at peak (under 0.5%), followed by a rapid decrease in shear resistance towards values of undrained steady state strength (Sus) as low as 100 psf or lower. The peak undrained strength is approximately equal to 0.3 times the consolidation stress.

The properties of the ash summarized above are consistent with the observation that the ash developed what is called a liquefaction failure, also referred to as a flow slide, during the December 2008 failure of the Kingston Dredge Cells. Because of its loose condition the ash is contractive, which means that it would tend to develop a decrease in volume when it is sheared. The shear strength of the saturated loose ash would depend on the degree of drainage that can occur as it is sheared. The strength referred to as "drained" would develop if the decrease in volume upon shearing actually occurs, which requires that the shearing be slow enough so that there will be enough time for the pore water in the ash to be expelled with minimum increase in pore pressure. Note that some pore pressure will develop no matter how slowly it is loaded since pore pressure is needed to cause a hydraulic gradient for the pore water to flow to a free boundary or to a pervious layer. If on the other hand shearing occurs faster than required for drainage, then there will not be enough time for the pore water to be expelled and the pore water pressure in the ash will increase substantially. The strength that develops in such a case is referred to as "undrained" strength. Because of the pore pressure development the undrained strength of the ash is substantially lower than its drained strength. The change from drained to undrained conditions leads to a large reduction in available strength and thus an earth structure that may be stable when the ash is sheared drained may become unstable when the ash is sheared undrained, which was the case for the Kingston Dredge Cells. Such a failure is termed a liquefaction failure. The change from drained to undrained behavior requires a triggering factor to cause the change.

There are several potential factors that have triggered known cases of liquefaction failures of saturated loose soils. In all cases the triggering factor is one that causes an increase in the rate of shearing in the loose soils. The most common one is seismic shaking and the failure is referred to as seismic liquefaction. There were no seismic events anywhere near the Kingston site that occurred within three days prior to the failure. The last train that run next to the dredge cells along the west edge of the dredge cells did so about 12 hours prior to the failure. Thus seismic or train induced vibrations are unlikely to have been the triggering event.

Liquefaction caused by non seismic triggering is referred to as static liquefaction. Static liquefaction has been reportedly due to factors which include a) slippage elsewhere in the soil mass that may cause a rapid transfer of shear stresses to the loose soils; b) an increase of the rate of loading; and c) local relatively rapid erosion at the toe of slopes or localized relatively small failures which cause a rapid increase in the shear stresses in the loose soils which did not participate in the small localized failure. The triggering event is often disproportionately small as compared with the magnitude of the liquefaction failure that follows, and thus in some cases the triggering factor is not apparent and the term "spontaneous" liquefaction has been used in the literature. Such a term is not intended to imply that the phenomenon can occur spontaneously, but that the triggering event was not identified.

The AECOM report reviews several potential triggering factors for the December 2008 failure at Kingston. We will discuss here only those that seem more likely.

An increase in the rate of loading is a potential triggering factor. The last stage of filling of cell 2 began about two months prior to the failure. At the northwest corner of cell 2, where the relic survey indicates that the failure started, the rate of loading was 6.1 ft per year, which is higher than in the previous active period in cell 2. However, in the phase 1 emergency cell the rate of loading was in the past more than twice than in the northwest corner of cell 2 prior to the failure (14.6 vs. 6.1 ft per year), and no failure developed in the phase 1 emergency cell even though the ash properties and the overall loading and configuration were similar to those in cell 2. Thus in my opinion, the increased rate of loading did not directly trigger the liquefaction of the ash.

Another potential triggering factor are localized failures, such as the shallow slides caused by seepage issues, which occurred in 2003 and 2006 along the slopes of the west dike of cell 2. Obviously the localized slides that did occur at those times did not cause a sufficiently large or sufficiently fast additional loading on the ash for it to go undrained, as no major failures ensued in 2003 and 2006. Furthermore, the seepage problems were addressed and inspections performed a short time prior to the failure indicated no seepage problems. Also the overall pattern of movements of the failure mass show that the west dikes moved to the north, not to the west, and that they appear to have moved over liquefied ash that was moving north from under the west dikes, which is consistent with the relic observations that indicate that the failure started at the north of the dredge cells and not along the west dikes. Thus in my opinion a repetition of localized failures similar to those observed in 2003 and 2006 along the west dikes were not the triggering mechanism for the liquefaction of the ash.

A TVA report of an inspection of the ash pond and dredge cells performed in October 20, 2008, about two months before the failure, noted that there was surficial erosion at several locations on dike slopes in areas lacking in vegetation. Such erosion appeared to be due to rainfall and in my opinion would cause neither a sufficiently large increase in stresses in the ash, nor a sufficiently rapid change in loading to cause static liquefaction.

Stability analysis performed at the NW corner of cell 2 indicate that the most critical failure surface would be one passing through the slimes and day lighting within the 200 ft setback area in the form of a wedge block. Assuming drained behavior in the ash and peak undrained strength in the slimes, the computed factor of safety ranges from 1.2 to 1.3, which is lower than computed elsewhere for the same assumptions, thus confirming the relic observations that indicate that the failure did start at the NW corner. Note that a key assumption inherent in stability analyses results in an overestimation of the factor of safety, which is particularly significant for the specific conditions at the Kingston site. The key assumption is that the peak strength can be mobilized simultaneously in all soils along the failure surface. The displacements required for the ash to mobilize its full drained strength are much higher than those required to mobilize the peak strength in the slimes. Given that the slime layer is thin, the 5% strain requires displacements of only 5% of its thickness that is at most 6 in, and thus the displacements to reach peak in the slimes are only of about 0.3 in or less. Such small displacements are too small to mobilize the drained strength of the slimes particularly in the passive side of the wedge block, which is located within the 200 ft setback area. Furthermore a creep test on the slimes show substantial creep at 80% of its peak strength and faster creep followed by failure at 85% of its peak strength, thus the peak strength of the slimes may not have an opportunity to develop. Thus the actual factor of safety was probably close to 1 just prior to the failure with the ash still behaving drained but the deformations accelerating as the slimes became undrained first and then were subject to creep, which triggered the ash to behave undrained and the consequent static liquefaction.

As the load increases and thus the mobilized drained strength in the ash becomes higher, the ash and other highly contractive soils become more susceptible to liquefaction, Castro (1994)[1]. The strain needed to reach the peak undrained strength, the pore pressure increase required to reach the peak strength and the additional shear stress to reach the

[1] Castro, G. "Seismically Induced Triggering of Liquefaction Failures", Performance of Ground and Soil Structures during Earthquakes, Thirteenth International Conference on Soil Mechanics and Foundation Engineering, New Delhi, India, 1994. (Copy attached to this letter).

peak strength, all become lower when the mobilized drained strength becomes higher. Thus the additional loading needed to trigger liquefaction became smaller as the height of the impoundment increased. At the same time the slimes became more susceptible to creep at the NW corner of cell 2. Thus the height of the impoundment in December of 2008 was a factor in contributing to the failure.

Note that the undrained behavior of the ash is extremely brittle, i.e. the ash collapses after a small shear strain (less than 0.5%) and then the available strength decreases rapidly with additional straining. Thus once even a small zone within the ash has turned to undrained behavior, there will be rapid shear stress increases propagated to the surrounding ash which in turn will go undrained and thus the change to undrained will rapidly propagate through the loose saturated ash. Furthermore, the peak undrained strength cannot be mobilized simultaneously throughout the ash because as it is mobilized in a zone that just turned undrained, in the adjacent zones the strain at peak would have been exceeded and the available resistance would have dropped below the peak and towards Sus. In geotechnical engineering this phenomenon is referred to as progressive failure, and it was an important factor in the static liquefaction of the ash.

Once the ash developed liquefaction the available strength of the ash became lower than the strength of the slimes and thus the subsequent series of failures that followed probably occurred within the ash and not through the slimes.

After careful consideration of the available information, I agree that the failure mechanism described in the AECOM report is the most probable. The mechanism consists of an initial failure at the NW corner through the slimes layer, which triggered static liquefaction of the ash. Liquefaction of the ash led to a sequence of subsequent slides. The first slide caused the failure of Dike C which was pushed to the north until it came to rest against the northern hill, followed by retrograde progressive liquefaction slides propagating southward until they reached a divider dike between cells 1 and 2. The height of the impoundment reached a critical height in December of 2008 that, given the overall configuration of the containment dikes, their slopes and setback from Dike C, lead to sufficient sliding on the slimes layer to induce the consequential liquefaction of the ash. The other potential triggering factors discussed in this letter are capable of triggering a static liquefaction failure, however in my opinion they were not likely to have been the triggering factor for the December 2008 failure.

We appreciate the opportunity to be of service to TVA and we will be pleased to answer any questions you might have concerning this letter.

Very truly yours

Gonzalo Castro

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